Ontario

Ministry of Transportation
Materials Engineering and Research Office Report

The Ministry of Transportation of Ontario's Experience With Electrochemical Chloride Extraction (ECE)

MERO- 001

Technical Report Documentation Page

Publication Title	**The Ministry of Transportation of Ontario's Experience with Electrochemical Chloride Extraction (ECE) 1989 - 2002**
Author(s)	Frank Pianca, Hannah Schell, David Lai, Rob Raven Ontario Ministry of Transportation
Originating Office	Concrete Section, Ontario Ministry of Transportation
Report Number	MERO-001; ISBN 0-7794-4100-1
Publication Date	January, 2003
Ministry Contact	Concrete Section, Ontario Ministry of Transportation 1201 Wilson Avenue, Room 235, Building C Downsview, Ontario, Canada M3M 1J8 **Tel: (416) 235-3705; Fax: (416) 235-5338** **Contact Person: F. Pianca, (416) 235-4691**
Abstract	The Ministry of Transportation of Ontario (MTO) first used ECE on an experimental basis in 1989. More recently, the Ministry has specified ECE on conventional contractor-bid contracts to mitigate the effect of corrosion distress on 5 piers and 1 abutment at the Burlington Bay Skyway in Burlington and 12 bridge abutments on Highway 11 in North Bay, Ontario. The paper will attempt to identify parameters that predict the probability of success of the ECE treatment. Typical costs experienced on the Ministry's ECE contracts to date will also be reported. The paper was submitted to the National Association of Corrosion Engineers for presentation at the Corrosion 2003 Conference. Based on the findings outlined in the report, the ECE process will soon be included in the Structure Rehabilitation Manual as a potential rehabilitation option for structures suffering from corrosion-induced distress.
Key Words	Corrosion, chloride ion, de-icing salts, highway bridges, reinforced concrete, rehabilitation, electrochemical chloride removal, ECR
Distribution	Unrestricted technical audience.

103-05545/

Ontario

Ministry of Transportation
Materials Engineering and Research Office Report

MERO-001

The Ministry of Transportation of Ontario's Experience With Electrochemical Chloride Extraction (ECE)

1989 - 2002

January 2003

Prepared by
Materials Engineering and Research Office
Concrete Section
Ontario Ministry of Transportation

1201 Wilson Avenue,
Downsview, Ontario, Canada M3M 1J8
Tel: (416) 235-3705; Fax (416) 235-5338

Published without prejudice as to the application of the findings.
Crown copyright reserved.

Table of Contents

Executive Summary ii

Introduction 1

Background ..... 2 - 7

Results ..... 8 - 10

Conclusions 11

References 12

Tables ..... 13 - 22

Figures ..... 23 - 29

Executive Summary

The corrosion of steel reinforcement in concrete highway structures involves electrochemical reactions and as such can be arrested by electrochemical means such as cathodic protection or electrochemical chloride extraction (ECE). The ECE method is typically similar to cathodic protection in that it involves application of current. It requires the installation of a temporary anode (5 - 8 weeks) on the surface of the concrete and the passage of direct current between the anode and the steel reinforcement which serves as the cathode. The current density of the ECE process is approximately 100 times the current density used in conventional cathodic protection.

The Ministry of Transportation of Ontario (MTO) first used ECE on an experimental basis in 1989. More recently, the Ministry has specified ECE on conventional contractor-bid contracts to mitigate the effect of corrosion distress on 5 piers and 1 abutment at the Burlington Bay Skyway in Burlington and 12 bridge abutments in North Bay, Ontario. Evaluation of ECE by the Ministry has indicated variable performance. Performance as expected, meeting the –250 mV static potential criterion specified by the Ministry has been documented on some structures but not on others. The reason for this variable performance is not yet well understood.

The paper will review variables such as total ampere hours passed, depth of concrete cover, depth of chloride corrosion threshold relative to the reinforcement, steel current density compared to concrete current density and corrosion potentials prior to and after treatment. Rate of corrosion measurements by the linear polarization technique will also be presented as well as concrete resistivity readings prior to and after treatment.

The paper will attempt to identify parameters that predict the probability of success of the ECE treatment. Typical costs experienced on the Ministry's ECE contracts to date will also be reported.

The paper was submitted to the National Association of Corrosion Engineers for presentation at the Corrosion 2003 Conference. Based on the findings outlined in the report, the ECE process will soon be included in the Structure Rehabilitation Manual as a potential rehabilitation option for structures suffering from corrosion-induced distress.

Introduction

Uncontaminated normal Portland cement provides excellent protection for steel reinforcement, both by acting as a barrier to corrosive ions and by providing a highly alkaline environment in which black steel becomes passivated and corrodes at a negligible rate. Concrete, however, contains a network of pores, a network of microcracks and, in some locations, continuous macrocracks. The protection due to the physical barrier is, therefore, imperfect and various ions penetrate the concrete cover by absorption and diffusion.

Chloride ions are considered to be the major cause of premature corrosion of steel reinforcement. Dissolved chloride ions may penetrate unprotected, hardened concrete in structures exposed to marine environments or to deicing salts.

The rate of corrosion of steel reinforcement embedded in concrete is strongly influenced by environmental factors. Both oxygen and moisture must be present if electrochemical corrosion is to occur. Reinforced concrete with significant gradients in chloride ion content is vulnerable to macro cell corrosion, especially if subjected to cycles of wetting and drying. Other factors that affect the rate and extent of corrosion are heterogeneities in the concrete and the steel, pH of the concrete pore water, carbonation of the Portland cement paste, cracks in the concrete, stray currents, and galvanic effects due to contact between dissimilar metals.

Deterioration of concrete due to corrosion results because the products of corrosion (rust) occupy a greater volume than the steel and exert substantial stresses on the surrounding concrete. The outward manifestations of the rusting include staining, cracking, and spalling of the concrete. Concurrently, the cross-section of the steel is reduced. With time, structural distress may occur either by loss of bond between the steel and concrete due to cracking and spalling or as a result of the reduced steel cross-sectional area.

Background

Theory and Application of Electrochemical Chloride Extraction (ECE)

When direct current is passed through an electrolyte, anions migrate toward the anode and cations migrate towards the cathode. This is the mechanism by which direct current is carried through concrete. All mobile ions present migrate under the influence of the electric field and, therefore, carry a portion of the current passed. In chloride-contaminated concrete, these ions include chloride ions and other ions typically found in concrete, including: hydroxyl, sodium, potassium and calcium ions. The amount of current carried by each type of ion species depends on several factors, including the total current passed, the mobility of the ion, the concentration of the ions present, and possibly temperature.

The process of electrochemical migration consists of applying an anode and electrolyte to the surface of a reinforced concrete structure and passing current between the anode and the reinforcement, which acts as the cathode. The technique is similar to cathodic protection but differs in two important respects: the anode is temporary and the current density is approximately 100 times that used in most cathodic protection installations.

Description

Burlington Bay Skyway – 1989 Trial

The Burlington Bay Skyway is a high level multi-span bridge constructed in 1955. In 1989 a 30.8 m^2 section near the base of one column in one of the piers (S-19) was treated using a proprietary ECE process. Three faces of the column were treated, the north face serving as a control. The reinforcement ratio was relatively light being 0.55 m^2 of steel surface for each 1 m^2 of concrete surface for the east and west faces, and 0.79 m^2 per m^2 of concrete surface for the north and south faces.

Prior to commencing the electrochemical treatment, a condition survey was made of the test site. This included:

- A visual survey of the concrete surface
- A delamination survey by sounding
- Measurement of concrete cover
- Verification of electrical continuity of the reinforcement
- Measurement of corrosion potentials (in accordance with ASTM)
- Corrosion rate measurements (using a commercial "3LP" device)
- Removal of cores for petrographic examination and measurement of chloride ion content.

The anode consisted of steel mesh encased in an electrolyte of cellulose fibre moistened with calcium hydroxide. The rectifier output was limited to 40 v. This resulted in an initial current density of 0.77 A/m^2 of concrete surface that had dropped to 0.26 A/m^2 at the end of the 8-week treatment period. The total charge passed was 610 A. HR/m^2 of concrete surface. At the conclusion of the treatment, the surface of the concrete was cleaned by blasting with an abrasive.

A more detailed description of the site installation, treatment and initial early results has been published elsewhere [1, 2].

Corrosion potential and rate of corrosion measurements were measured periodically using a copper – copper sulfate half cell and a commercial three-electrode polarization device,

on the surface of the concrete. The long-term (9 year) results are summarized in Tables 1 and 2.

Burlington Bay Skyway - 1997

In the summer of 1997 ECE was applied to the south abutment (70 m^2) and to Pier S34 (188 m^2) of the Burlington Bay Skyway under Contract 20-97-412 using the same proprietary system used in 1989. This represented the first time the Ministry specified ECE using routine contracting and tendering procedures. The acceptance criteria for the treatment required that the treatment process would be run until one of the following conditions existed:

a) The chloride in the vicinity of the reinforcing steel is less than or equal to 0.03% by weight of concrete (after correcting for background chlorides).

b) The half-cell potential (versus Cu-$CuSO_4$), after treatment and sufficient time for depolarization, shall not be more negative than –250 mV measured at any location on the concrete surface.

c) A total of 700 A. HR/m^2 (concrete surface area) of current has been passed.

Prior to the application of ECE, a 1989 consultant's condition survey (Table 3) found that Pier S34 had 9% spalls and delaminations. The south abutment had 1% spalls and delaminations. ECE was selected because the structure reinforcement was corroding but was determined not to be in an advanced state of corrosion.

The half-cell potentials measured prior to treatment, and 7, 20, 24 and 54 months after treatment are shown in Table 4.

The average linear polarization prior to treatment, at 7 and 54 months after treatment, are shown in Table 5. The commercial "3LP" instrument used to evaluate corrosion currents by the linear polarization technique was the National Bureau of Standards model.

Measured concrete cover on the ECE treated structures is shown in Table 6. Acid soluble chloride ion distribution prior to and after treatment is shown in Figures 1 - 5.

Power was applied to the south abutment on September 4, 1997. The rectifier voltage was limited to 40 volts. The initial current density was approximately 0.5 A/m^2 of concrete surface. The current density declined with time. The power was turned off on October 29, 1997; after 55 days of treatment the current density at that time was approximately 0.19 A/m^2. The south abutment and pier S34 received approximately 421 and 378 A. HR/m^2 of current respectively based on concrete surface.

Burlington Bay Skyway - 1999

The same proprietary ECE system used in 1989 was subsequently used to rehabilitate 4 piers on the substructure of the Burlington Bay in Burlington, Ontario during the summer/fall of 1999 (Figures 6 and 7). This project (Contract 99-25) involved the treatment of 4 piers, with a total area of approximately 1500 m^2 (S19 – 372 m^2, S22 – 609 m^2, S26 – 287 m^2, S30 – 238 m^2).

The ECE treatment required the installation of a temporary anode on the surface of the piers. Three of the piers used a steel mesh anode and the fourth (S-30) used a titanium mesh anode. The treatment lasted from 6 to 8 weeks.

The project consisted of conventional chip and patch repairs on all existing spalled and delaminated areas, followed by ECE to all 4 piers. The conventional repairs were completed by the general contractor and the proprietary ECE treatment was completed by a subcontractor (Vector Restoration Limited).

In June 2000 (approximately 7 months after ECE treatment was completed) a consultant performed a corrosion potential survey on the 4 piers of the Burlington Skyway Bridge. The structure potentials were measured with a portable $Cu/CuSO_4$ reference electrode placed directly on the concrete surface of the pier. The potential measurements were located on a grid system of dimensions 1 m by 1 m for Piers S19 and S30 and dimensions of 2 m by 2 m for Piers S22 and S26. The readings correspond in location to readings prior to the ECE treatment. Before and after potential readings are shown in Tables 7 and 8. Readings taken by Ministry personnel were also obtained 3 years after treatment.

North Bay – 2000

ECE was used to rehabilitate 7 structures on Highway 11 from Lakeshore Drive northerly to the CNR overpass in North Bay, Ontario during the summer/fall of 2000 (Figures 8 and 9). The project (Contract 2000-210) involved the treatment of 12 abutments, 4 wing-walls and 2 retaining walls. Only the performance of the abutments will be reported on. The overall combined concrete area of ECE treatment for all 7 structures was approximately 696 m^2.

ECE was chosen as the rehabilitation option because previous condition surveys indicated that the abutments demonstrated high corrosion potentials with only a few delaminations or spalls. The treatment of each structure lasted approximately 8 weeks. Again the project consisted of conventional chip and patch repairs on all existing spalled and delaminated areas, followed by the application of ECE. The conventional repairs were completed by the general contractor and the ECE treatment was completed by a subcontractor experienced in ECE technology/application.

As part of the contract, a corrosion potential survey was conducted (by a consultant) on all abutment walls prior to commencing with the ECE treatment. The Ministry conducted a corrosion potential survey to assess the ECE treatment both in 2001 and 2002. As part of the 2001 study, 1 core was extracted from each abutment face (for acid soluble chloride analysis). All cores were taken in corresponding high corrosion potential areas, between the reinforcing bars. The corrosion condition of the abutments prior to the application of ECE and 1 and 2 years after application are shown in Tables 9 and 10.

Cost Analysis

ECE was applied to the substructure of the Burlington Bay Skyway in 2 different phases. During each phase a general contractor was successful in bidding for the work (generally the low bidder). The general contractor would then be responsible for the majority of the work (staging, concrete removals, patching etc). However, if the general contractor was not familiar with ECE they would then hire a sub contractor experienced with the technology (a licensed applicator).

The cost for the application of the ECE at Burlington (charged by the subcontractor to the general contractor) included costs for the mobilization, monitoring and system operation for the ECE plus profits. This cost was approximately $\$250/m^2$. However, the final cost charged to the Ministry was about $\$500/m^2$. Reasons for the price increase could be attributed to the general contractor applying a safety risk factor for not being familiar with the technology, the perception of difficulty with the relatively new technology and profits. Actual costs reported by the ECE applicator may not be reduced over time due to a lack of competition resulting from a single source application.

As a cost comparison, concrete refacing (including 100% concrete removal below reinforcing bars) generally costs $\$450/m^2$. Cathodic protection with its rectifier and associated wiring and permanent anodes costs approximately $\$250/m^2$. However, this cost does not include the continuous monitoring and maintenance required for cathodic protection systems.

Costs for the ECE in North Bay evaluated by the Bridge Office and reported in an internal Ministry document confirm the high price of ECE similar to that experienced at the Burlington Bay Skyway.

For cathodic protection to be specified as a rehabilitation option, the Ministry requires that the treated area be greater than 500 m^2. The reason for this is that based on experience, the cost associated with the installation, monitoring and maintenance of cathodic protection is generally not justified for smaller treated areas. Similarly, a size restriction may apply to ECE since the extra cost for mobilization, monitoring and system operation must be offset by cost savings realized because chloride contaminated concrete (based on half cell) will not be required to be removed.

Other possible solutions to the cost issue may include using an itemized tender (similar to the system used for cathodic protection) or having the ECE work bid directly to the Ministry.

Results

Pier S-19, 1989

In 1989, ECE was applied as an experiment to a 30.8 m^2 section of Pier S-19. As can be seen from Tables 1 and 2, the treatment was successful in passivating the reinforcement for 9 years. The readings remained passive throughout that period with no indication or trend indicating a return to more corrosive values. In 1999, ECE was applied to the entire pier; as a result, readings on the experimental section were not continued.

In 1989, a total charge of 610 A HR/m^2 of concrete surface was passed. Due, in large part, to the good long-term performance experienced at S-19, ECE was also applied to 5 piers and 1 abutment at the Burlington Bay Skyway.

Burlington Bay 1997

In order to ensure good performance from the ECE process, the Ministry specified the following acceptance criteria.

The final acceptance was based on applying the treatment process until one of the following conditions existed:

1. The chloride in the vicinity of the reinforcing steel after treatment is less than or equal to 0.03% by weight of concrete (after correcting for background chlorides).

2. The half-cell potential (versus Cu – $CuSO_4$) after treatment and allowing sufficient time for depolarization (usually 6 months) shall not be more negative than –250 mV measured at any location on the concrete surface.

3. A total of 700 A. HR/m^2 (concrete surface area) of current has been passed. In subsequent trials after 1997 this number was reduced to 600 A. HR/m^2.

Using the second criterion, all of the piers at Burlington (treated in 1997 and 1999) have performed well (100% compliance may not be practical or attainable; arbitrarily, the authors have selected a maximum 15% non compliance as acceptable). Although the corrosion performance of the south abutment at Burlington has greatly improved after application of ECE, it has not performed as well as the piers (Table 8). The reason for this may be due to the fact that the concrete at the abutment is in contact with soil and in general contain more moisture than concrete at the piers. Perhaps a greater portion of the current is carried by the hydroxyl ions instead of the chloride ions.

The rate of corrosion by the linear polarization technique for Pier S34 and the south abutment is reported in Table 5. Interpretation of the readings is as follows: if readings are less than 0.2 $\mu a/cm^2$, no corrosion is expected; if readings are between 0.2 and 1.0 $\mu a/cm^2$ corrosion is possible in 10 to 15 years; if readings are between 1.0 and 10 $\mu a/cm^2$, corrosion is expected in 2 to 10 years; if readings are greater than 10 $\mu a/cm^2$, corrosion is expected in 2 years or less. The rate of corrosion readings confirms that the corrosion currents have been greatly reduced after application of the ECE and have remained so for 54 months.

Concrete resistivity readings were measured on Pier S34 and the south abutment prior to and after ECE application using a Wenner four-point probe. The readings indicate that there is a significant increase in the concrete resistivity after ECE application and this increase was consistent and present in the latest readings (5 years after application).

During the application of ECE, a strong electrical field is generated that causes ions to migrate – positive ions in one direction and negative ions in the other direction. At some point, some ions may combine to form compounds that tend to increase the resistivity of the concrete. The compounds generally are less mobile than the ions. Also, the compounds may block the concrete pores and capillaries again with the effect of increasing concrete resistivity. This secondary effect of ECE is considered positive since higher resistivity concrete is generally associated with reduced corrosion currents.

Burlington 1999

In 1999, 4 piers were treated with ECE. The final results (Tables 7 and 8) showed good results in that the reinforcement was rendered passive (met the potential requirement). Three of the 4 piers used a mild steel anode but on Pier S-30 a titanium anode was used. Good performance was achieved by the inert anode plus it had the added advantage that after treatment, the abrasive blasting required to remove the unsightly rust stains on the concrete surface was not required. This may lead to potential cost savings.

North Bay 2000

Using the corrosion potential requirement, most of the 12 abutments (9 of 12) treated in North Bay did not perform as well as expected. Performance could be related to a number of factors. The original data showed that large areas of the abutments demonstrated active corrosion. Many abutments had areas equal to or greater than 40% in this range. This is an indication that the chloride ion corrosion threshold may have penetrated well beyond the top reinforcement level.

As previously mentioned, abutment concrete tends to be wetter; therefore, ECE may not be as efficient on abutments. Chloride analysis of the cores removed from the abutments (after ECE treatment by the Ministry) indicated that the corrected (for background) chloride levels were generally above threshold at the level of the reinforcement.

Treatment of the abutments in North Bay was continued into the fall. In northern Ontario, the temperature at this time of the year can be quite cool. The efficiency of the ECE treatment becomes less efficient with time. Combining this with the cooler air temperature, which would tend to increase the resistivity of the concrete, the final performance of the ECE may have been compromised.

The total passage of current, from an end user's point of view, needs to be better defined. Current based on concrete surface is preferred. The reason for this is that a structure, depending on location, may have several steel to concrete ratios. Current density based on concrete surface area would allow an easier determination if the target (ideal) current charge of 600 A-HR/m^2 (based on the original work at S-19) was achieved. As an example, the steel/concrete ratio varied from 0.07 to 0.77 in the treated areas at North Bay. The main reason for not using steel area is due to the lengthy and subjective process of determining the area from structural drawings.

However, it should be noted that significant benefits to the abutments were still realized by the application of the ECE treatment. From Table 9, there is significant corrosion reduction due to the ECE treatment if the –0.35 v ASTM criterion is used.

Concrete cover over the reinforcement is reported in Table 6. On all structures the concrete cover is good, generally greater than 60 mm. From Ministry experience, structures with good concrete cover generally perform better than structures with poor concrete cover. The reason for this may be explained as by the time corrosion distress has been identified on structures with poor cover (less than 37 mm) the chlorides may have reached deep beneath the top mat of reinforcement making removal of chloride ions away from the reinforcement more difficult.

The North Bay ECE contract required the treatment of a number of abutments located at several different sites. Many of the abutments had relatively small areas. These conditions may not result in the most economical use of the ECE.

Conclusions

1. Based on the work at the Burlington Bay Skyway, significant long-term passivation of the reinforcement can be achieved by the ECE treatment. ECE is an effective technique for extending the corrosion-free service life of reinforced concrete structures, when applied appropriately.

2. ECE may not be as efficient on structures with high moisture content or on structures where the chloride ion threshold has penetrated well beyond the top mat of reinforcement.

3. Since ECE is in competition with other rehabilitation methods, the cost of applying ECE must be reduced, in order for the treatment to be considered for specification on a routine basis. It is felt that lack of contractor familiarity with this new technique, and a single source of availability contributed to the high costs experienced to date.

4. Inconsistency in measurement of total charge passed during ECE may compromise the assessment of the effectiveness of the treatment. It is recommended that a standard method of expressing total charge passed be used, reported with respect to concrete surface area.

References

1. Manning, D. G., and Pianca, F., "Electrochemical Removal of Chloride Ions from Reinforced Concrete: Initial Evaluation of the Pier S19 Field Trial", Transportation Research Board No. 1304, pages 153 – 159 (1991).

2. Manning, D. G., and Ip, A. K. C., "Rehabilitating Corrosion-Damaged Bridges Through the Electrochemical Migration of Chloride Ions", Ministry of Transportation of Ontario, 1994.

3. Broomfield, J. P., "Corrosion of Steel in Concrete, Understanding, Investigation and Repair", E. & FN SPON.

4. Broomfield, J. P, and Buenfeld, N. R., "The Effect of Electrochemical Chloride Extraction (ECE) on Concrete Properties", Paper No. 97 0464 Transportation Research Board, January 1997.

5. Clemena, G. G., and Jackson, R. D., "Pilot Applications of Electrochemical Chloride Extraction on Concrete Bridge Decks in Virginia", Paper No. 97-0300, Transportation Research Board, January 1997.

6. Nustad, G. E., "Norcure Desalination – Electrochemical Technology to Arrest Chloride-Induced Corrosion in Concrete", Paper No. 97-1491, Transportation Research Board, January 1997.

7. Beaudette, M. R., "Norcure Electrochemical Chloride Extraction Project, MTO Contract #2000-210, North Bay, Ontario, "Vector Construction Group, July 2000 – Jan ry 2001.

Table 1: Pier S-19 (1989)

Corrosion Potential Measurements

Percentage of Readings Within Range

Corrosion Potentials (mV vs. Cu-CuS04, negative sign omitted)

Time After Treatment	North			West			South			East		
	<200	200-350	>350	<200	200-350	>350	<200	200-350	>350	<200	200-350	>350
Original	0	85	15	0	96	4	0	96	4	0	84	16
6 Weeks	0	81	19	38	42	20	22	33	45	29	31	40
6 Months	63	37	0	96	4	0	100	0	0	98	2	0
10 Months	30	70	0	96	4	0	100	0	0	98	2	0
13 Months	41	59	0	98	2	0	100	0	0	100	0	0
20 Months	30	70	0	98	2	0	100	0	0	93	7	0
25 Months	41	59	0	100	0	0	100	0	0	98	2	0
35 Months	26	74	0	96	4	0	100	0	0	96	4	0
48 Months	26	70	4	98	2	0	100	0	0	96	4	0
60 Months	19	74	7	96	4	0	96	4	0	96	4	0
72 Months	26	59	15	96	4	0	93	7	0	93	7	0
84 Months	30	63	7	96	4	0	96	4	0	91	9	0
96 Months	11	78	11	96	4	0	93	7	0	87	13	0
108 Months	15	78	7	96	4	0	93	7	0	87	13	0

Table 2: Corrosion Current Measurements for Pier S19 (1989)

	Average Corrosion Current (uA/cm^2)			
	North	**West**	**South**	**East**
Original	1.24	0.71	0.60	1.61
6 Weeks	0.90	0.15	0.11	0.16
6 Months	0.81	0.22	0.10	0.17
13 Months	1.24	0.15	0.10	0.18
25 Months	0.75	0.15	0.08	0.26
35 Months	0.57	0.11	0.06	0.14
48 Months	0.54	0.10	0.04	0.17
60 Months	1.49	0.29	0.17	0.37
72 Months	1.69	0.21	0.18	0.18
84 Months	1.01	0.20	0.15	0.20
96 Months	1.64	0.19	0.17	0.27

Table 3: Summary of Consultant's 1989 Survey Showing Deterioration and Probability of Corrosion

Pier No.	Delam./Spalls %	< -0.35 V %	-0.20 V to -0.35 V %	> -0.20 V %
S19	8	37	55	8
S22	4	14	63	23
S26	10	25	69	6
S30	9	22	70	8
S34	9	10	72	18
S. Abut.	1	16	56	28

Table 4: Corrosion Potentials Before and After ECE of Pier S34 and South Abutment (1997)

Location	Number of Months After ECE	<200 mV %	200 mV – 350 mV %	>350 mV %
South Abutment	Before	18	64	18
	7 Months	58	54	0
	20 Months	75	25	0
	24 Months	80	20	0
	54 Months	58	42	0
S34 – South Elevation	Before	28	63	9
	7 Months	93	7	0
	20 Months	96	4	0
	24 Months	100	0	0
	54 Months	100	0	0
S34 – North Elevation	Before	21	63	16
	7 Months	93	7	0
	20 Months	96	4	0
	24 Months	100	0	0
	54 Months	100	0	0

Table 5: Linear Polarization Results for South Abutment and Pier S34 (1997)

Pier No.	Average Corrosion Current (ua/cm^2)		
	Before ECE	7 Months After ECE	54 Months After ECE
S Abutment	2.14	0.77	0.72
S34	1.99	N/A	0.73

Table 6: Summary of ECE Data (1997, 1999 and 2000)

No.	Location	Concrete Area m^2	Cover mm	Current Passed Based on Steel Area
1	South Abutment	70	76	421*
2	S34	188	93	378*
3	S30	238	87	1408
4	S26	287	98	1026
5	S22	609	94	667
6	S19	372	96	956
7	Lavasse (SBL) South Abutment	46	66	745
8	Lavasse (SBL) North Abutment	36	67	1017
9	Lavasse (NBL) South Abutment	41	62	942
10	Lavasse (NBL) North Abutment	32	60	924
11	Lakeshore Drive West Abutment	45	59	881
12	Lakeshore Drive East Abutment	45	63	887
13	Birch's Rd. (NBL) South Abutment	32	48	846
14	Birch's Rd. (SBL) North Abutment	30	56	785
15	Birch's Rd. (SBL) South Abutment	34	64	1311
16	Birch's Rd. (SBL) North Abutment	29	65	985
17	CNR SBL South Abutment	51	72	1017
18	CNR SBL North Abutment	48	66	1041

*** Based on concrete surface**

Table 7: Results of Half Cell Testing Before and After 1999 Treatment of Piers S19, S22, S26 and S30

Pier No.	Number of Years After ECE	<200 mV %	200 mV – 350 mV %	>350 mV %
S19	Before	13	73	14
	1 Year	97	3	0
	3 Years	100	0	0
S22	Before	38	39	23
	1 Year	90	9	1
	3 Years	88	12	0
S26	Before	7	91	2
	1 Year	96	4	0
	3 Years	100	0	0
S30	Before	36	64	0
	1 Year	100	0	0
	3 Years	100	0	0

Table 8: Percentage of Readings ≤ -0.26 V (wrt $Cu/CuS0_4$) (1997and 1999)

Location	Time		
	1 Year After %	3 Years After %	5 Years After %
South Abutment	19	N/A	11
S34 South Elevation	0	N/A	0
S34 North Elevation	0	N/A	0
S30	0	0	N/A
S26	2	0	N/A
S22	2	2	N/A
S19	1	0	N/A

Table 9: North Bay – 2000 Electrochemical Chloride Extraction Results

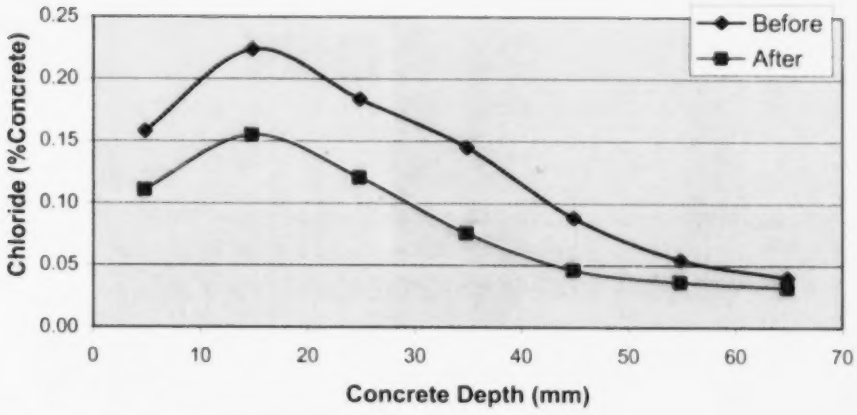
No.	Structure	Before ECE %			1 Year After ECE %			2 Years After ECE %		
		0 to -0.19	-0.20 to -0.35	< -350	0 to -0.19	-0.20 to -0.35	< -350	0 to -0.19	-0.20 to -0.35	< -350
1	Lavasse R./CPR O'Hd. (SBL) South Abutment	2	30	68	18	80	2	0	92	8
2	Lavasse R./CPR O'Hd. (SBL) North Abutment	0	3	97	39	61	0	2	90	8
3	Lavasse R./ CPR O'Hd. (NBL) South Abutment	0	7	93	4	96	0	0	88	12
4	Lavasse R./ CPR O'Hd. (NBL) North Abutment	0	7	93	17	83	0	0	96	4
5	Lakeshore Drive U.P. West Abutment	0	20	80	2	92	6	0	94	7
6	Lakeshore Drive U.P. East Abutment	0	38	62	16	84	0	0	100	0
7	Birch's Rd. O.P. (NBL) South Abutment	11	53	36	23	77	0	10	90	0
8	Birch's Rd. O.P. (NBL) North Abutment	11	75	14	22	78	0	18	79	3
9	Birch's Rd. O.P. (SBL) South Abutment	11	69	20	66	34	0	44	56	0
10	Birch's Rd. O.P. (SBL) North Abutment	47	50	3	97	3	0	62	38	0
11	CNR O.H. (SBL) South Abutment	6	18	76	32	68	0	2	94	4
12	CNR O.H. (SBL) South Abutment	10	44	46	49	51	0	0	98	2

Table 10: North Bay – 2000 ECE Results

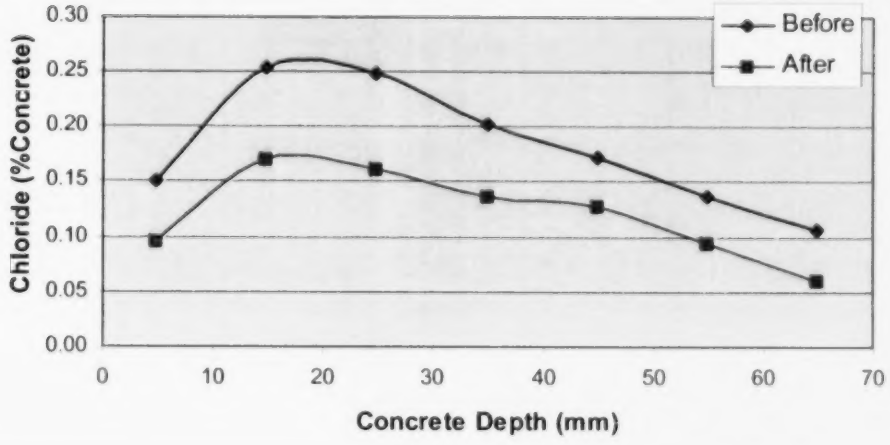
Table 8: Percentage of Readings ≤ -0.26 V (wrt $Cu/CuS0_4$)

No.	Structure	Percentage of Readings ≤ -0.260	
		1 Year After ECE	2 Years After ECE
1	Lavasse R./CPR O'Hd. (SBL) South Abutment	47	73
2	Lavasse R./CPR O'Hd. (SBL) North Abutment	27	73
3	Lavasse R./CPR O'Hd. (NBL) South Abutment	71	98
4	Lavasse R./CPR O'Hd. (NBL) North Abutment	31	73
5	Lakeshore Drive U.P. West Abutment	80	95
6	Lakeshore Drive U.P. East Abutment	29	69
7	Birch's Rd. O.P. (NBL) South Abutment	5	13
8	Birch's Rd. O.P. (NBL) North Abutment	11	51
9	Birch's Rd. O.P. (SBL) South Abutment	8	13
10	Birch's Rd. O.P. (SBL) North Abutment	0	8
11	CNR O.H. (SBL) South Abutment	33	53
12	CNR O.H. (SBL) North Abutment	15	53

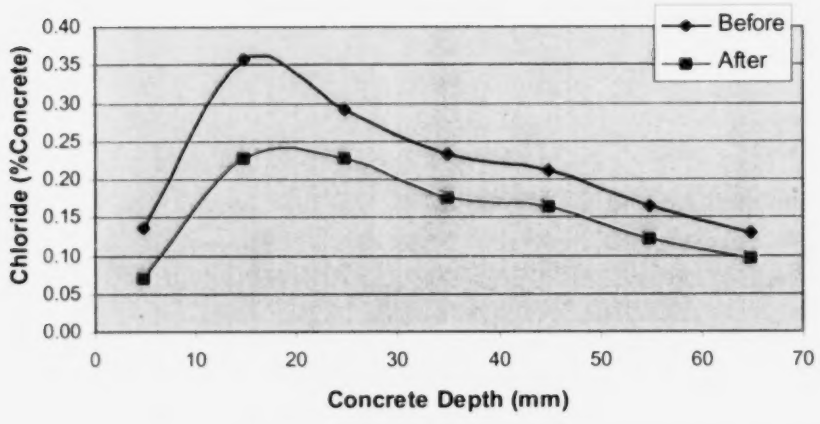
Burlington Bay ECE (1997)
South Abutment Core 1- (Figure 1)



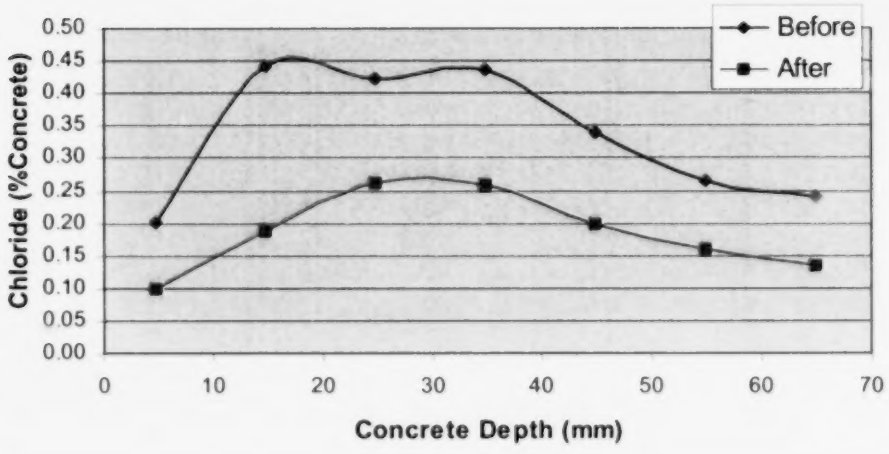
Burlington Bay ECE (1997)
South Abutment Core 3- (Figure 2)



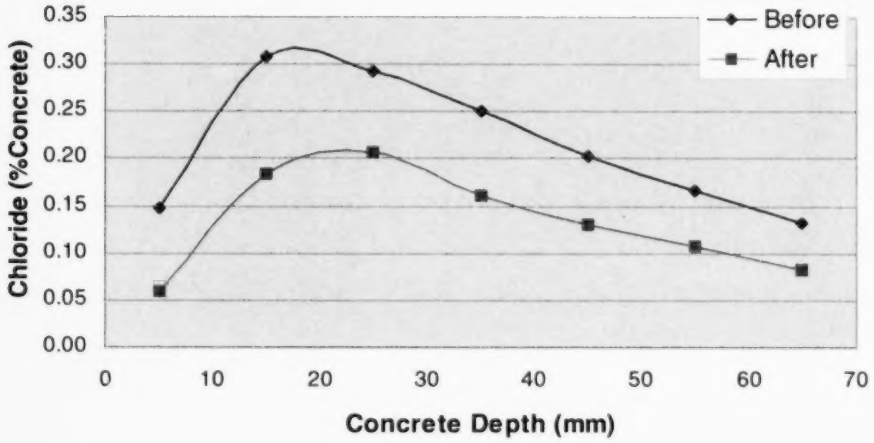
Burlington Bay ECE (1997)
West Column South Face- (Figure 3)



Burlington Bay ECE (1997)
West Column North Face- (Figure 4)



Burlington Bay ECE (1997)
East Column South Face- (Figure 5)



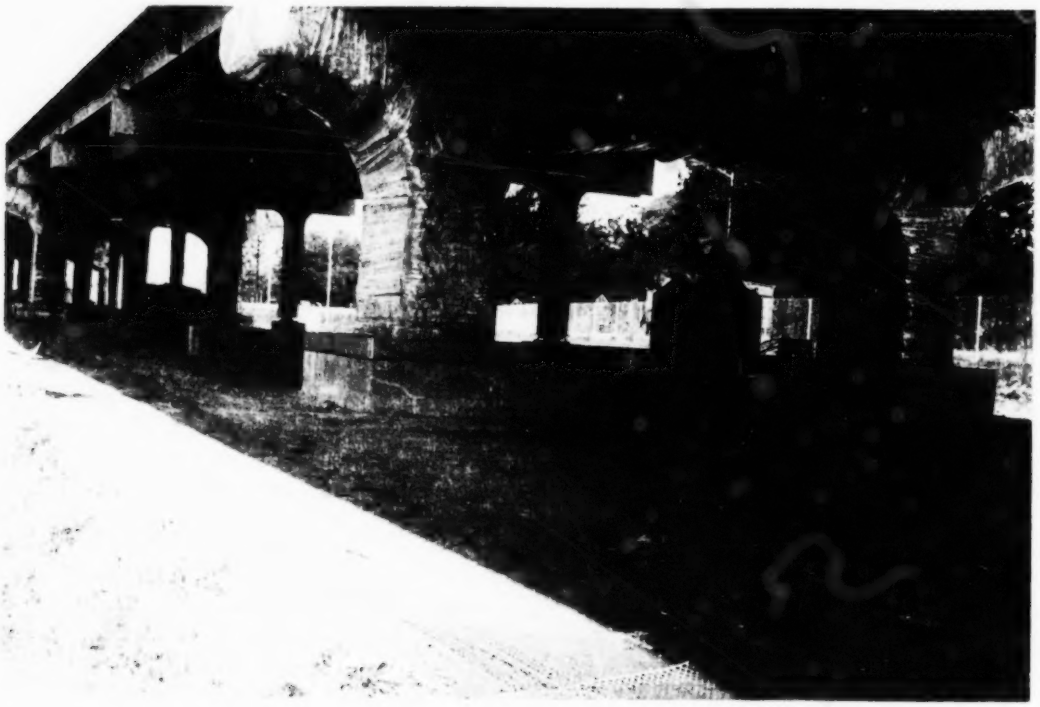


Figure 6 – General View of Application of ECE
Burlington Skyway, 1999

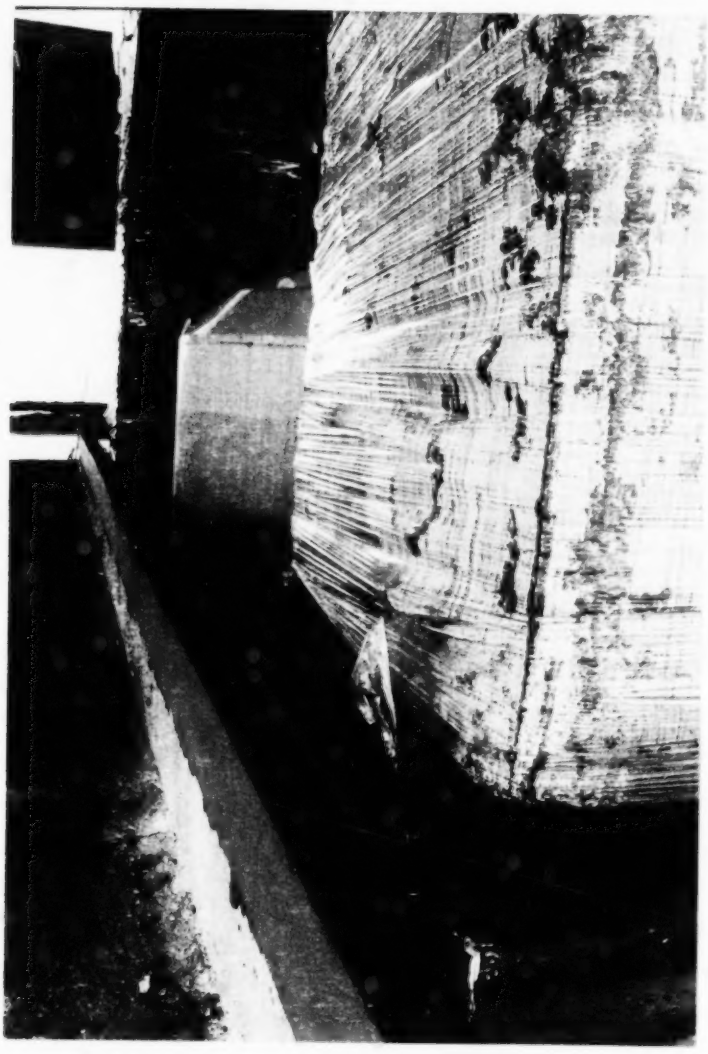


Figure 7 – Close-up Showing Collection Set-up for Excess Electrolyte

Figure 8 - Lakeshore Drive, North Bay
East Abutment Face

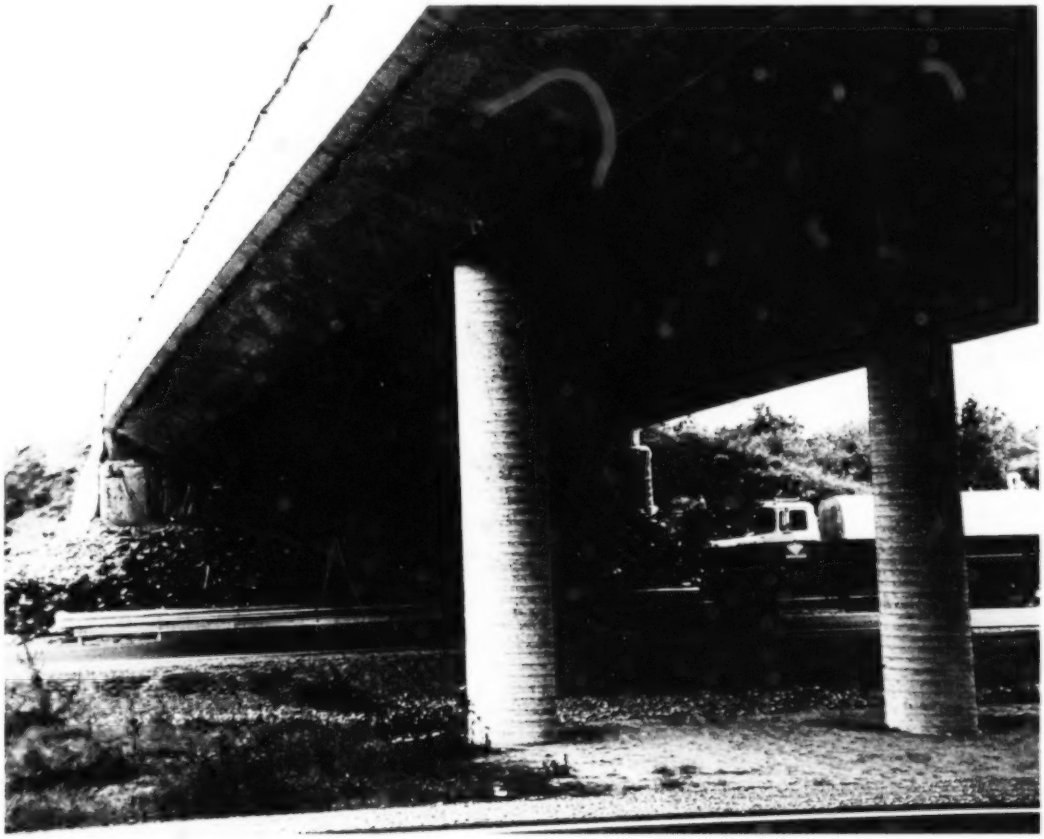


Figure 9 - Lakeshore Drive, North Bay
West Abutment Face